Chapter 2
Functional Design

2-1. Shoreline Use

Some structures are better suited than others for particular shoreline uses. Revetments of randomly placed stone may hinder access to a beach, while smooth revetments built with concrete blocks generally present little difficulty for walkers. Seawalls and bulkheads can also create an access problem that may require the building of stairs. Bulkheads are required, however, where some depth of water is needed directly at the shore, such as for use by boaters.

2-2. Shoreline Form and Composition

a. Bluff shorelines. Bluff shorelines that are composed of cohesive or granular materials may fail because of scour at the toe or because of slope instabilities aggravated by poor drainage conditions, infiltration, and reduction of effective stresses due to seepage forces. Cantilevered or anchored bulkheads can protect against toe scour and, being embedded, can be used under some conditions to prevent sliding along subsurface critical failure planes. The most obvious limiting factor is the height of the bluff, which determines the magnitude of the earth pressures that must be resisted, and, to some extent, the depth of the critical failure surface. Care must be taken in design to ascertain the relative importance of toe scour and other factors leading to slope instability. Gravity bulkheads and seawalls can provide toe protection for bluffs but have limited applicability where other slope stability problems are present. Exceptions occur in cases where full height retention is provided for low bluffs and where the retained soil behind a bulkhead at the toe of a higher bluff can provide sufficient weight to help counterbalance the active thrust of the bluff materials.

b. Beach shorelines. Revetments, seawalls, and bulkheads can all be used to protect backshore developments along beach shorelines. As described in paragraph 1-4*c*, an important consideration is whether wave reflections may erode the fronting beach.

2-3. Seasonal Variations of Shoreline Profiles

Beach recession in winter and growth in summer can be estimated by periodic site inspections and by computed variations in seasonal beach profiles. The extent of winter beach profile lowering will be a contributing factor in determining the type and extent of needed toe protection.

2-4. Design Conditions for Protective Measures

Structures must withstand the greatest conditions for which damage prevention is claimed in the project plan. All elements must perform satisfactorily (no damage exceeding ordinary maintenance) up to this condition, or it must be shown that an appropriate allowance has been made for deterioration (damage prevention adjusted accordingly and rehabilitation costs amortized if indicated). As a minimum, the design must successfully withstand conditions which have a 50 percent probability of being exceeded during the project's economic life. In addition, failure of the project during probable maximum conditions should not result in a catastrophe (i.e., loss of life or inordinate loss of money).

2-5. Design Water Levels

The maximum water level is needed to estimate the maximum breaking wave height at the structure, the amount of runup to be expected, and the required crest elevation of the structure. Minimum expected water levels play an important role in anticipating the amount of toe scour that may occur and the depth to which the armor layer should extend.

a. Astronomical tides. Changes in water level are caused by astronomical tides with an additional possible component due to meteorological factors (wind setup and pressure effects). Predicted tide levels are published annually by the National Oceanic and Atmospheric Administration (NOAA). The statistical characteristics of astronomical tides at various U.S. ports were analyzed in Harris (1981) with probability density functions of water levels summarized in a series of graphs and tables. Similar tables are available for the Atlantic Coast in Ebersole (1982) which also includes estimates of storm surge values.

b. Storm surge. Storm surge can be estimated by statistical analysis of historical records, by methods described in Chapter 3 of the Shore Protection Manual (SPM), or through the use of numerical models. The numerical models are usually justified only for large projects. Some models can be applied to open coast studies, while others can be used for bays and estuaries where the effects of inundation must be considered.

c. Lake levels. Water levels on the Great Lakes are subject to both periodic and nonperiodic changes. Records dating from 1836 reveal seasonal and annual changes due to variations in precipitation. Lake levels (particularly Ontario and Superior) are also partially controlled by regulatory works operated jointly by Canadian and U.S. authorities. These tend to minimize water level variations in those lakes. Six-month forecasts of lake levels are published monthly by the Detroit District (Figure 2-1).

2-6. Design Wave Estimation

Wave heights and periods should be chosen to produce the most critical combination of forces on a structure with due consideration of the economic life, structural integrity, and hazard for events that may exceed the design conditions (see paragraph 2-4). Wave characteristics may be based on an analysis of wave gauge records, visual observations of wave action, published wave hindcasts, wave forecasts, or the maximum breaking wave at the site. Wave characteristics derived from such methods may be for deepwater locations and must be transformed to the structure site using refraction and diffraction techniques as described in the SPM. Wave analyses may have to be performed for extreme high and low design water levels and for one or more intermediate levels to determine the critical design conditions.

2-7. Wave Height and Period Variability and Significant Waves

a. Wave height.

(1) A given wave train contains individual waves of varying height and period. The significant wave height, H_s, is defined as the average height of the highest one-third of all the waves in a wave train. Other wave heights such as H_{10} and H_1 can also be designated, where H_{10} is the average of the highest 10 percent of all waves, and H_1 is the average of the highest 1 percent of all waves. By assuming a Rayleigh distribution, it can be stated that

$$H_{10} \approx 1.27\, H_s \tag{2-1}$$

and

$$H_1 \approx 1.67\, H_s \tag{2-2}$$

(2) Available wave information is frequently given as the energy-based height of the zeroth moment, H_{mo}. In deep water, H_s and H_{mo} are about equal; however, they may be significantly different in shallow water due to shoaling (Thompson and Vincent 1985). The following equation may be used to equate H_s from energy-based wave parameters (Hughes and Borgman 1987):

$$\frac{H_s}{H_{mo}} = \exp\left[C_0\left(\frac{d}{g\,T_p^2}\right)^{-C_1}\right] \tag{2-3}$$

where

C_0, C_1 = regression coefficients given as 0.00089 and 0.834, respectively

d = water depth at point in question (i.e., toe of structure)

g = acceleration of gravity

T_p = period of peak energy density of the wave spectrum

A conservative value of H_s may be obtained by using 0.00136 for C_0, which gives a reasonable upper envelope for the data in Hughes and Borgman. Equation 2-3 should not be used for

$$\frac{d}{g\,T_p^2} < 0.0005 \tag{2-4}$$

or where there is substantial wave breaking.

(3) In shallow water, H_s is estimated from deepwater conditions using the irregular wave shoaling and breaking model of Goda (1975, 1985) which is available as part of the Automated Coastal Engineering System (ACES) package (Leenknecht et al. 1989). Goda (1985) recommends for the design of rubble structures that if the depth is less than one-half the deepwater significant wave height, then design should be based on the significant wave height at a depth equal to one-half the significant deepwater wave height.

b. Wave period. Wave period for spectral wave conditions is typically given as period of the peak energy density of the spectrum, T_p. However, it is not uncommon to find references and design formulae based on the average wave period (T_z) or the significant wave period (T_s , average period of the one-third highest waves). Rough guidance on the relationship among these wave periods is given in Table 2.1.

c. Stability considerations. The wave height to be used for stability considerations depends on whether the

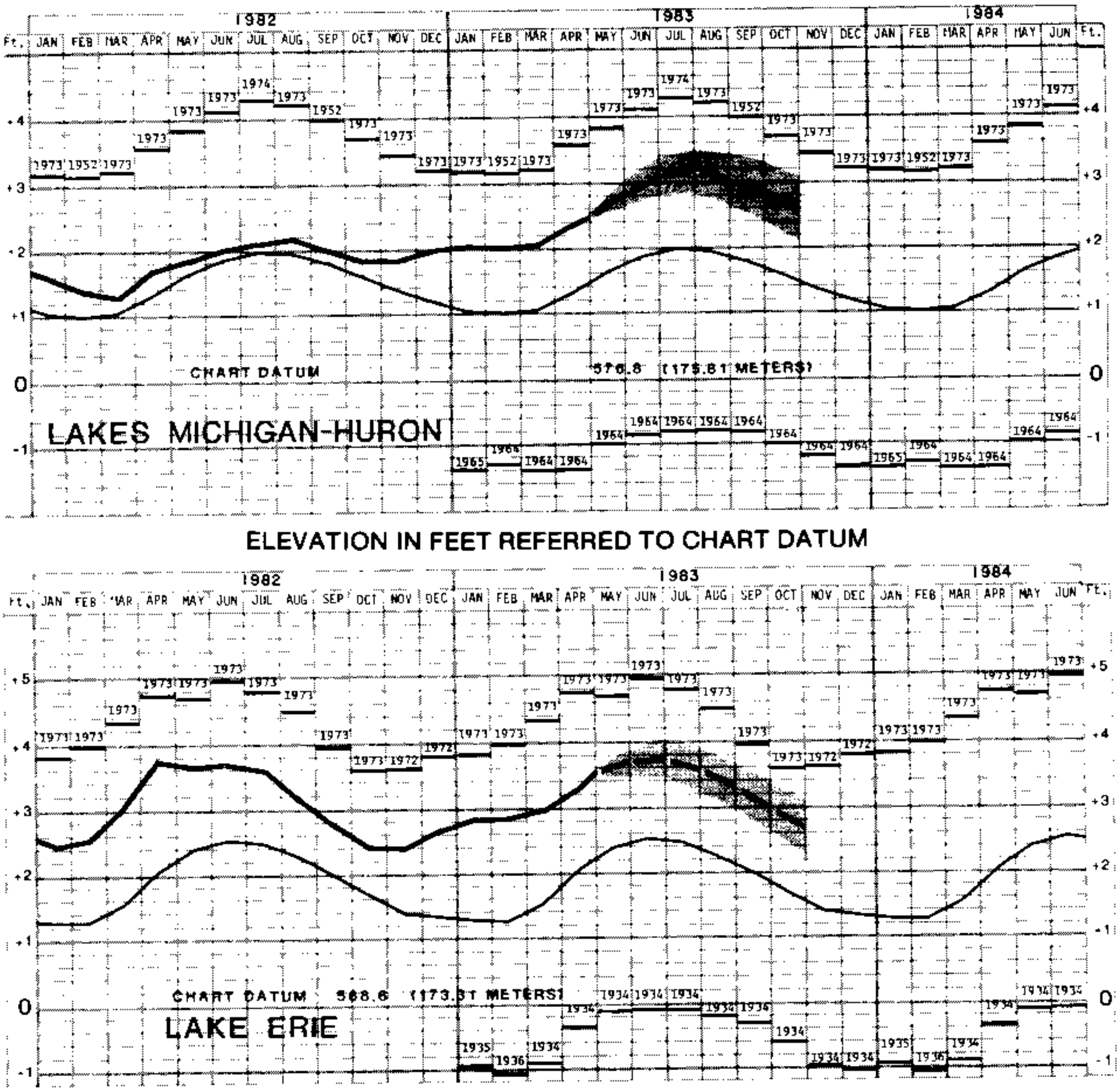

Figure 2-1. Monthly lake level forecast

structure is rigid, semirigid, or flexible. Rigid structures that could fail catastrophically if overstressed may warrant design based on H_1. Semirigid structures may warrant a design wave between H_1 and H_{10}. Flexible structures are usually designed for H_s or H_{10}. Stability coefficients are coupled with these wave heights to develop various degrees of damage, including no damage.

2-8. Wave Gauges and Visual Observations

Available wave data for use by designers is often sparse and limited to specific sites. In addition, existing gauge data are sometimes analog records which have not been analyzed and that are difficult to process. Project funding

Table 2-1
Relationships among T_p, T_s, and T_z

T_z / T_p	T_s / T_p	Comments	γ
0.67	0.80	Severe surf zone conditions[1]	NA
0.74	0.88	Pierson-Moskowitz spectrum[2]	1.0
0.80	0.93	Typical JONSWAP spectrum[2]	3.3
0.87	0.96	Swell from distant storms[2]	10.0

[1] Developed from data in Ahrens (1987).
[2] Developed from Goda (1987).

and time constraints may prohibit the establishment of a viable gauging program that would provide sufficient digital data for reliable study. Visual observations from shoreline points are convenient and inexpensive, but they have questionable accuracy, are often skewed by the omission of extreme events, and are sometimes difficult to extrapolate to other sites along the coast. A visual wave observation program is described in Schneider (1981). Problems with shipboard observations are similar to shore observations.

2-9. Wave Hindcasts

Designers should use the simple hindcasting methods in ACES (Leenknecht et al. 1989) and hindcasts developed by the U.S. Army Engineer Waterways Experiment Station (WES) (Resio and Vincent 1976-1978; Corson et al. 1981) for U.S. coastal waters using numerical models. These later results are presented in a series of tables for each of the U.S. coasts. They give wave heights and periods as a function of season, direction of wave approach, and return period; wave height as a function of return period and seasons combined; and wave period as a function of wave height and approach angle. Several other models exist for either shallow or deep water. Specific applications depend on available wind data as well as bathymetry and topography. Engineers should stay abreast of developments and choose the best method for a given analysis. Contact the Coastal Engineering Research Center (CERC) at WES for guidance in special cases.

2-10. Wave Forecasts

Wave forecasts can be performed using the same methodologies as those for the wave hindcasts. Normally, the Corps hindcasts waves for project design, and the Navy forecasts waves to plan naval operations.

2-11. Breaking Waves

a. Wave heights derived from a hindcast should be checked against the maximum breaking wave that can be supported at the site given the available depth at the design still-water level and the nearshore bottom slope. Figure 2-2 (Weggel 1972) gives the maximum breaker height, H_b, as a function of the depth at the structure, d_s , nearshore bottom slope, m, and wave period, T. Design wave heights, therefore, will be the *smaller* of the maximum breaker height or the hindcast wave height.

b. For the severe conditions commonly used for design, H_{mo} may be limited by breaking wave conditions. A reasonable upper bound for H_{mo} is given by

$$(H_{mo})_{\max} = 0.10\, L_p \tanh\left(\frac{2\pi d}{L_p}\right) \tag{2-5}$$

where L_p is wavelength calculated using T_p and d.

2-12. Height of Protection

When selecting the height of protection, one must consider the maximum water level, any anticipated structure settlement, freeboard, and wave runup and overtopping.

2-13. Wave Runup

Runup is the vertical height above the still-water level (swl) to which the uprush from a wave will rise on a structure. Note that it is not the distance measured along the inclined surface.

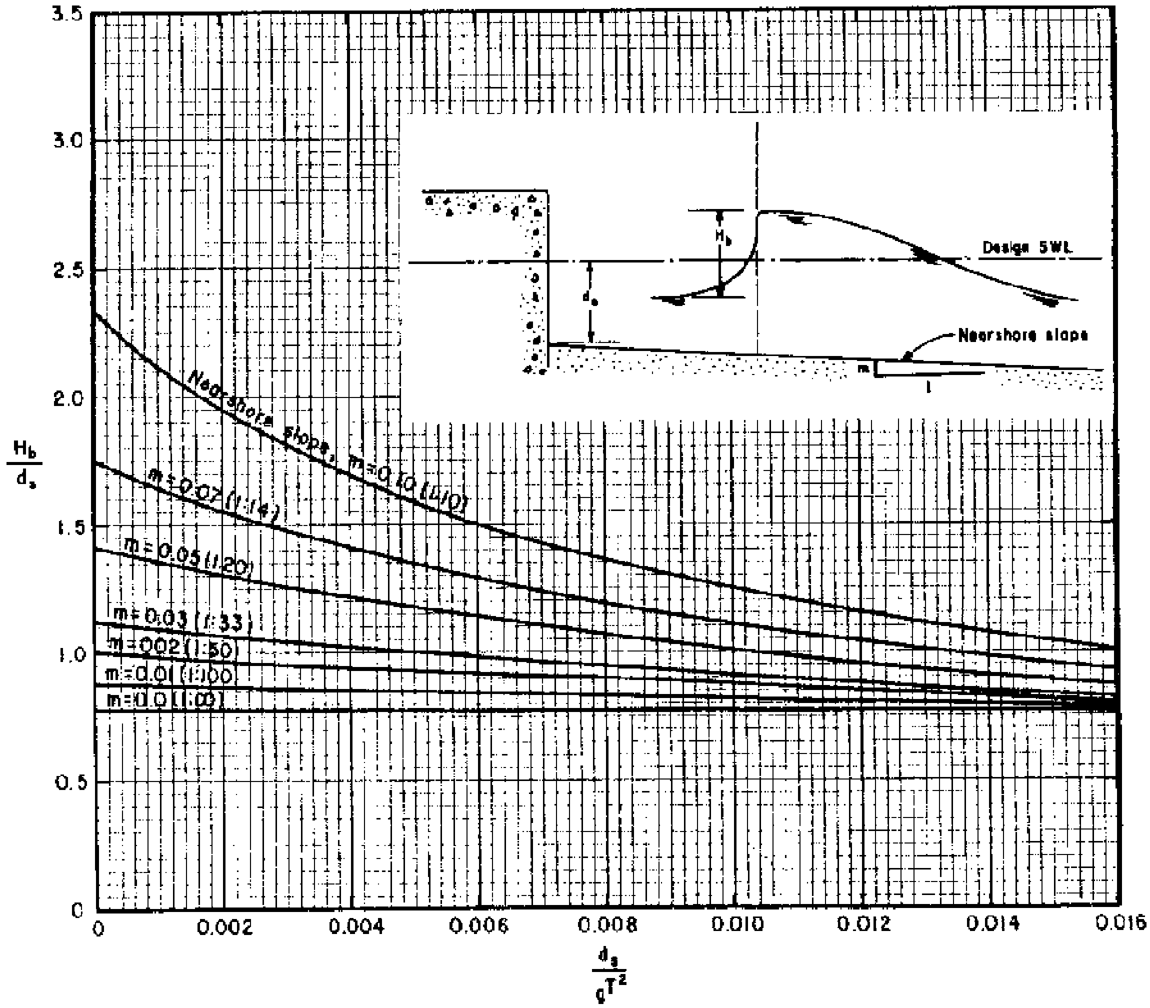

Figure 2-2. Design breaker height

a. Rough slope runup.

(1) Maximum runup by irregular waves on riprap-covered revetments may be estimated by (Ahrens and Heimbaugh 1988)

$$\frac{R_{max}}{H_{mo}} = \frac{a\xi}{1 + b\xi} \qquad (2\text{-}6)$$

where

R_{max} = maximum vertical height of the runup above the swl

a, b = regression coefficients determined as 1.022 and 0.247, respectively

ξ = surf parameter defined by

$$\xi = \frac{\tan\theta}{\left(\dfrac{2\pi H_{mo}}{gT_p^2}\right)^{1/2}} \qquad (2\text{-}7)$$

where θ is the angle of the revetment slope with the horizontal. Recalling that the deepwater wavelength may be determined by

$$L_o = \frac{g T_p^2}{2\pi} \tag{2-8}$$

the surf parameter is seen to be the ratio of revetment slope to square root of wave steepness. The surf parameter is useful in defining the type of breaking wave conditions expected on the structure, as shown in Figure 2-3.

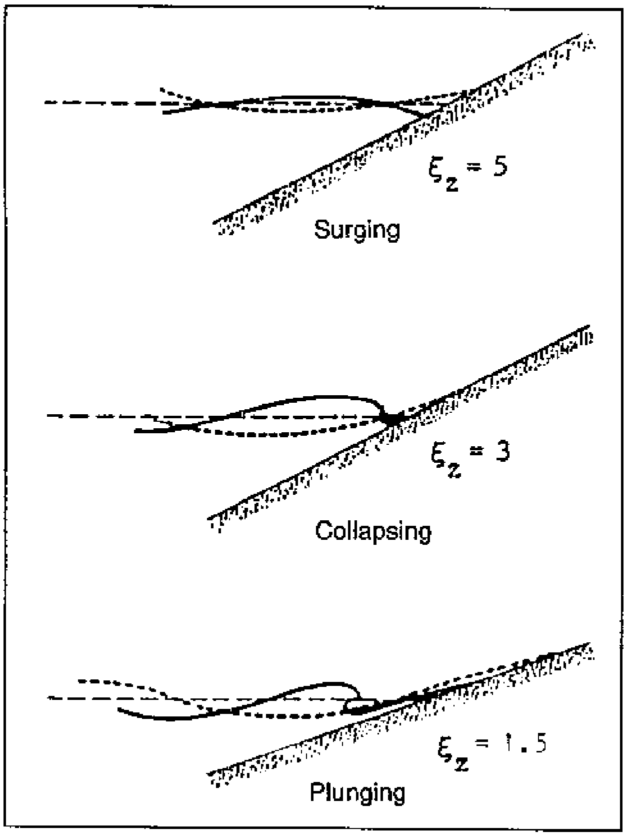

Figure 2-3. Surf parameter and breaking wave types

(2) A more conservative value for R_{max} is obtained by using 1.286 for a in Equation 2-6. Maximum runups determined using this more conservative value for a provide a reasonable upper limit to the data from which the equation was developed.

(3) Runup estimates for revetments covered with materials other than riprap may be obtained with the rough slope correction factors in Table 2-2. Table 2-2 was developed for earlier estimates of runup based on monochromatic wave data and smooth slopes. To use the correction factors in Table 2-2 with the irregular wave rough slope runup estimates of Equation 2-6, multiply R_{max} in Equation 2-6 by the correction factor listed in Table 2-2, and divide by the correction factor for quarrystone. For example, to estimate R_{max} for a stepped 1:1.5 slope with vertical risers, determine R_{max} by Equation 2-6 and multiply by (correction factor for stepped slope/correction factor for quarrystone) (0.75/0.60) = 1.25. R_{max} for the stepped slope is seen to be 25 percent greater than for a riprap slope.

b. Smooth slope runup. Runup values for smooth slopes may be found in design curves in the SPM. However, the smooth slope runup curves in the SPM were based on monochromatic wave tests rather than more realistic irregular wave conditions. Using H_s for wave height with the design curves will yield runup estimates that may be exceeded by as much as 50 percent by waves in the wave train with heights greater than H_s. Maximum runup may be estimated by using Equation 2-6 and converting the estimate to smooth slope by dividing the result by the quarrystone rough slope correction factor in Table 2-2.

c. Runup on walls. Runup determinations for vertical and curved-face walls should be made using the guidance given in the SPM.

2-14. Wave Overtopping

a. It is generally preferable to design shore protection structures to be high enough to preclude overtopping. In some cases, however, prohibitive costs or other considerations may dictate lower structures than ideally needed. In those cases it may be necessary to estimate the volume of water per unit time that may overtop the structure.

b. Wave overtopping of riprap revetments may be estimated from the dimensionless equation (Ward 1992)

$$Q' = C_0 e^{C_1 F'} e^{C_2 m} \tag{2-9}$$

where Q' is dimensionless overtopping defined as

$$Q' = \frac{Q}{\left(g H_{mo}^3\right)^{1/2}} \tag{2-10}$$

where Q is dimensional overtopping in consistent units, such as cfs/ft. F' in Equation 2-9 is dimensionless freeboard defined as

Table 2-2
Rough Slope Runup Correction Factors (Carstea et al. 1975b)

Armor Type	Slope (cot θ)	Relative Size H / K_r[a,b]	Correction Factor r
Quarrystone	1.5	3 to 4	0.60
Quarrystone	2.5	3 to 4	0.63
Quarrystone	3.5	3 to 4	0.60
Quarrystone	5	3	0.60
Quarrystone	5	4	0.68
Quarrystone	5	5	0.72
Concrete Blocks[c]	Any	6[b]	0.93
Stepped slope with vertical risers	1.5	$1 \le H_o'/K_r$[d]	0.75
Stepped slope with vertical risers	2.0	$1 \le H_o'/K_r$[d]	0.75
Stepped slope with vertical risers	3.0	$1 \le H_o'/K_r$[d]	0.70
Stepped slope with rounded edges	3.0	$1 \le H_o'/K_r$[d]	0.86
Concrete Armor Units			
Tetrapods random two layers	1.3 to 3.0	-	0.45
Tetrapods uniform two layers	1.3 to 3.0	-	0.51
Tribars random two layers	1.3 to 3.0	-	0.45
Tribars uniform one layer	1.3 to 3.0	-	0.50

[a] K_r is the characteristic height of the armor unit perpendicular to the slope. For quarrystone, it is the nominal diameter; for armor units, the height above the slope.
[b] Use H_o' for $d_s/H_o' > 3$; and the local wave height, H_s for $d_s/H_o' \le 3$.
[c] Perforated surfaces of Gobi Blocks, Monoslaps, and concrete masonry units placed hollows up.
[d] K_r is the riser height.

$$F' = \frac{F}{\left(H_{mo}^2 L_o\right)^{1/3}} \tag{2-11}$$

where F is dimensional freeboard (vertical distance of crest above swl). The remaining terms in Equation 2-9 are m (cotangent of revetment slope) and the regression coefficients C_0, C_1, and C_2 defined as

$$\begin{aligned} C_0 &= 0.4578 \\ C_1 &= -29.45 \\ C_2 &= 0.8464 \end{aligned} \tag{2-12}$$

The coefficients listed above were determined for dimensionless freeboards in the range $0.25 < F' < 0.43$, and revetment slopes of 1:2 and 1:3.5.

c. Overtopping rates for seawalls are complicated by the numerous shapes found on the seawall face plus the variety of fronting berms, revetments, and steps. Information on overtopping rates for a range of configurations is available in Ward and Ahrens (1992). For bulkheads and simple vertical seawalls with no fronting revetment and a small parapet at the crest, the overtopping rate may be calculated from

$$Q' = C_0 \exp\left[C_1 F' + C_2\left(\frac{F}{d_s}\right)\right] \tag{2-13}$$

where Q' is defined in Equation 2-10, F' is defined in Equation 2-11, d_s is depth at structure toe, and the regression coefficients are defined by

$$\begin{aligned} C_0 &= 0.338 \\ C_1 &= -7.385 \\ C_2 &= -2.178 \end{aligned} \tag{2-14}$$

For other configurations of seawalls, Ward and Ahrens (1992) should be consulted, or physical model tests should be performed.

2-15. Stability and Flexibility

Structures can be built by using large monolithic masses that resist wave forces or by using aggregations of smaller units that are placed either in a random or in a well-ordered array. Examples of these are large reinforced concrete seawalls, quarrystone or riprap revetments, and geometric concrete block revetments. The massive monoliths and interlocking blocks often exhibit superior initial strength but, lacking flexibility, may not accommodate small amounts of differential settlement or toe scour that may lead to premature failure. Randomly placed rock or concrete armor units, on the other hand, experience settlement and readjustment under wave attack, and, up to a point, have reserve strength over design conditions. They typically do not fail catastrophically if minor damages are inflicted. The equations in this chapter are suitable for preliminary design for major structures. However, final design will usually require verification of stability and performance by hydraulic model studies. The design guidance herein may be used for final design for small structures where the consequences of failure are minor. For those cases, project funds are usually too limited to permit model studies.

2-16. Armor Unit Stability

a. The most widely used measure of armor unit stability is that developed by Hudson (1961) which is given in Equation 2-15:

$$W = \frac{\gamma_r H^3}{K_D \left(\frac{\gamma_r}{\gamma_w} - 1 \right)^3 \cot \theta} \tag{2-15}$$

where

W = required individual armor unit weight, lb (or W_{50} for graded riprap)

γ_r = specific weight of the armor unit, lb/ft^3

H = monochromatic wave height

K_D= stability coefficient given in Table 2-3

γ_w = specific weight of water at the site (salt or fresh)

θ = is structure slope (from the horizontal)

Stones within the cover layer can range from 0.75 to 1.25 W as long as 50 percent weigh at least W and the gradation is uniform across the structure's surface. Equation 2-15 can be used for preliminary and final design when H is less than 5 ft and there is no major overtopping of the structure. For larger wave heights, model tests are preferable to develop the optimum design. Armor weights determined with Equation 2-15 for monochromatic waves should be verified during model tests using spectral wave conditions.

b. Equation 2-15 is frequently presented as a stability formula with N_s as a stability number. Rewriting Equation 2-15 as

$$N_s = \frac{H}{\left(\frac{W}{\gamma_r} \right)^{1/3} \left(\frac{\gamma_r}{\gamma_w} - 1 \right)} \tag{2-16}$$

it is readily seen that

$$N_s = \left(K_D \cot \theta \right)^{1/3} \tag{2-17}$$

By equating Equations 2-16 and 2-17, W is readily obtained.

c. For irregular wave conditions on revetments of dumped riprap, the recommended stability number is

$$N_{sz} = 1.14 \cot^{1/6} \theta \tag{2-18}$$

where N_{sz} is the zero-damage stability number, and the value 1.14 is obtained from Ahrens (1981b), which recommended a value of 1.45 and using H_s with Equation 2-16, then modified based on Broderick (1983), which found using H_{10} (10 percent wave height, or average of highest 10-percent of the waves) in Equation 2-16 provided a better fit to the data. Assuming a Rayleigh wave height distribution, $H_{10} \approx 1.27\ H_s$. Because H_s is more readily available than H_{10}, the stability number in Equation 2-17 was adjusted (1.45/1.27 = 1.14) to allow H_s to be used in the stability equation while providing the more conservative effect of using H_{10} for the design.

d. Stability equations derived from an extensive series of laboratory tests in The Netherlands were presented in van der Meer and Pilarczyk (1987) and van der

Table 2-3
Suggested Values for Use In Determining Armor Weight (Breaking Wave Conditions)

Armor Unit	n[1]	Placement	Slope (cot θ)	K_D
Quarrystone				
Smooth rounded	2	Random	1.5 to 3.0	1.2
Smooth rounded	>3	Random	1.5 to 3.0	1.6
Rough angular	1	Random	1.5 to 3.0	Do Not Use
Rough angular	2	Random	1.5 to 3.0	2.0
Rough angular	>3	Random	1.5 to 3.0	2.2
Rough angular	2	Special[2]	1.5 to 3.0	7.0 to 20.0
Graded riprap[3]	2[4]	Random	2.0 to 6.0	2.2
Concrete Armor Units				
Tetrapod	2	Random	1.5 to 3.0	7.0
Tripod	2	Random	1.5 to 3.0	9.0
Tripod	1	Uniform	1.5 to 3.0	12.0
Dolos	2	Random	2.0 to 3.0[5]	15.0[6]

[1] n equals the number of equivalent spherical diameters corresponding to the median stone weight that would fit within the layer thickness.

[2] Special placement with long axes of stone placed perpendicular to the slope face. Model tests are described in Markle and Davidson (1979).

[3] Graded riprap is not recommended where wave heights exceed 5 ft.

[4] By definition, graded riprap thickness is two times the diameter of the minimum W_{50} size.

[5] Stability of dolosse on slope steeper than 1 on 2 should be verified by model tests.

[6] No damage design (3 to 5 percent of units move). If no rocking of armor (less than 2 percent) is desired, reduce K_D by approximately 50 percent.

Meer (1988a, 1988b). Two stability equations were presented. For plunging waves,

$$N_s = 6.2\, P^{0.18} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \xi_z^{0.5} \tag{2-19}$$

and for surging or nonbreaking waves,

$$N_s = 1.0\, P^{-0.13} \left(\frac{S}{\sqrt{N}} \right)^{0.2} \sqrt{\cot\theta}\; \xi_z^{P} \tag{2-20}$$

where

P = permeability coefficient

S = damage level

N = number of waves

P varies from P = 0.1 for a riprap revetment over an impermeable slope to P = 0.6 for a mound of armor stone with no core. For the start of damage S = 2 for revetment slopes of 1:2 or 1:3, or S = 3 for revetment slopes of 1:4 to 1:6. The number of waves is difficult to estimate, but Equations 2-19 and 2-20 are valid for N = 1,000 to N = 7,000, so selecting 7,000 waves should provide a conservative estimate for stability. For structures other than riprap revetments, additional values of P and S are presented in van der Meer (1988a, 1988b).

e. Equations 2-19 and 2-20 were developed for deepwater wave conditions and do not include a wave-height truncation due to wave breaking. van der Meer therefore recommends a shallow water correction given as

$$\frac{N_{s\ (shallow\ water)}}{N_{s\ (deep\ water)}} = \frac{1.40\, H_s}{H_2} \tag{2-21}$$

where H_2 is the wave height exceeded by 2 percent of the waves. In deep water, $H_2 \approx 1.40\ H_s$, and there is no correction in Equation 2-21.

2-17. Layer Thickness

a. Armor units. As indicated in the SPM, the thickness of an armor layer can be determined by Equation 2-22:

$$r = n\,k_{\Delta}\left(\frac{W}{w_r}\right)^{1/3} \tag{2-22}$$

where r is the layer thickness in feet, n is the number of armor units that would fit within the layer thickness (typically n=2), and k_{Δ} is the layer coefficient given in Table 2-4. For estimating purposes, the number of armor units, N_r, for a given surface area in square feet, A, is

$$N_r = A\,n\,k_{\Delta}\left(1-\frac{P}{100}\right)\left(\frac{w_r}{W}\right)^{\frac{2}{3}} \tag{2-23}$$

where P is the average porosity of the cover layer from Table 2-4.

b. Graded riprap. The layer thickness for graded riprap must be at least twice the nominal diameter of the W_{50} stone, where the nominal diameter is the cube root of the stone volume. In addition, $r_{\min}$ should be at least 25 percent greater than the nominal diameter of the largest stone and should always be greater than a minimum layer thickness of 1 ft (Ahrens 1975). Therefore,

$$r_{\min} = \max\left[2.0\left(\frac{W_{50\,\min}}{\gamma_r}\right)^{1/3};\; 1.25\left(\frac{W_{100}}{\gamma_r}\right)^{1/3};\; 1\ ft\right] \tag{2-24}$$

where $r_{\min}$ is the minimum layer thickness perpendicular to the slope. Greater layer thicknesses will tend to increase the reserve strength of the revetment against waves greater than the design. Gradation (within broad limits) appears to have little effect on stability provided the W_{50} size is used to characterize the layer. The following are suggested guidelines for establishing gradation limits (from EM 1110-2-1601) (see also Ahrens 1981a):

(1) The lower limit of W_{50} stone, $W_{50\,\min}$, should be selected based on stability requirements using Equation 2-15.

(2) The upper limit of the W_{100} stone, $W_{100\,\max}$, should equal the maximum size that can be economically obtained from the quarry but not exceed 4 times $W_{50\,\min}$.

(3) The lower limit of the W_{100} stone, $W_{100\,\min}$, should not be less than twice $W_{50\,\min}$.

(4) The upper limit of the W_{50} stone, $W_{50\,\max}$, should be about 1.5 times $W_{50\,\min}$.

(5) The lower limit of the W_{15} stone, $W_{15\,\min}$, should be about 0.4 times $W_{50\,\min}$.

(6) The upper limit of the W_{15} stone, $W_{15\,\max}$, should be selected based on filter requirements specified in EM 1110-2-1901. It should slightly exceed $W_{50\,\min}$.

(7) The bulk volume of stone lighter than $W_{15\,\min}$ in a gradation should not exceed the volume of voids in the revetment without this lighter stone. In many cases, however, the actual quarry yield available will differ from the gradation limits specified above. In those cases the designer must exercise judgment as to the suitability of the supplied gradation. Primary consideration should be given to the $W_{50\,\min}$ size under those circumstances. For instance, broader than recommended gradations may be suitable if the supplied W_{50} is somewhat heavier than the required $W_{50\,\min}$. Segregation becomes a major problem, however, when the riprap is too broadly graded.

2-18. Reserve Stability

a. General. A well-known quality of randomly placed rubble structures is the ability to adjust and resettle under wave conditions that cause minor damages. This has been called reserve strength or reserve stability. Structures built of regular or uniformly placed units such as concrete blocks commonly have little or no reserve stability and may fail rapidly if submitted to greater than design conditions.

b. Armor units. Values for the stability coefficient, K_D, given in paragraph 2-16 allow up to 5 percent damages under design wave conditions. Table 2-5 contains values of wave heights producing increasing levels of damage. The wave heights are referenced to the zero-damage wave height ($H_{D=0}$) as used in Equation 2-15. Exposure of armor sized for $H_{D=0}$ to these larger wave heights should produce damages in the range given. If the armor stone available at a site is lighter than the stone size calculated using the wave height at the site, the zero-damage wave height for the available stone can be

Table 2-4
Layer Coefficients and Porosity for Various Armor Units

Armor Unit	n	Placement	K_Δ	P (%)
Quarrystone (smooth)	2	Random	1.00	38
Quarrystone (rough)	2	Random	1.00	37
Quarrystone (rough)	≥3	Random	1.00	40
Graded riprap	2[a]	Random	N/A	37
Tetrapod	2	Random	1.04	50
Tribar	2	Random	1.02	54
Tribar	1	Uniform	1.13	47
Dolos	2	Random	0.94	56

[a] By definition, riprap thickness equals two cubic lengths of W_{50} or 1.25 W_{100}.

Table 2-5
$H/H_{D=0}$ for Cover Layer Damage Levels for Various Armor Types ($H/H_{D=0}$ for Damage Level in Percent)

Unit	$0 \le \%_D < 5$	$5 \le \%_D < 10$	$10 \le \%_D < 15$	$15 \le \%_D < 20$	$20 \le \%_D \le 30$
Quarrystone (smooth)	1.00	1.08	1.14	1.20	1.29
Quarrystone (angular)	1.00	1.08	1.19	1.27	1.37
Tetrapods	1.00	1.09	1.17	1.24	1.32
Tribars	1.00	1.11	1.25	1.36	1.50
Dolos	1.00	1.10	1.14	1.17	1.20

calculated, and a ratio with the site's wave height can be used to estimate the damage that can be expected with the available stone. All values in the table are for randomly placed units, n=2, and minor overtopping. The values in Table 2-5 are adapted from Table 7-8 of the SPM. The SPM values are for breakwater design and nonbreaking wave conditions and include damage levels above 30 percent. Due to differences in the form of damage to breakwaters and revetments, revetments may fail before damages reach 30 percent. The values should be used with caution for damage levels from breaking and non-breaking waves.

c. Graded riprap. Information on riprap reserve stability can be found in Ahrens (1981a). Reserve stability appears to be primarily related to the layer thickness although the median stone weight and structure slope are also important.

2-19. Toe Protection

a. General. Toe protection is supplemental armoring of the beach or bottom surface in front of a structure which prevents waves from scouring and under-cutting it. Factors that affect the severity of toe scour include wave breaking (when near the toe), wave runup and backwash, wave reflection, and grain-size distribution of the beach or bottom materials. The revetment toe often requires special consideration because it is subjected to both hydraulic forces and the changing profiles of the beach fronting the revetment. Toe stability is essential because failure of the toe will generally lead to failure throughout the entire structure. Specific guidance for toe design based on either prototype or model results has not been developed. Some empirical suggested guidance is contained in Eckert (1983).

b. Revetments.

(1) Design procedure. Toe protection for revetments is generally governed by hydraulic criteria. Scour can be caused by waves, wave-induced currents, or tidal currents. For most revetments, waves and wave-induced currents will be most important. For submerged toe stone, weights can be predicted based on Equation 2-25:

$$W_{\min} = \frac{\gamma_r H^3}{N_s^3 \left(\frac{\gamma_r}{\gamma_w} - 1 \right)^3} \tag{2-25}$$

where N_s is the design stability number for rubble toe protection in front of a vertical wall, as indicated in the SPM (see Figure 2-7). For toe structures exposed to wave action, the designer must select either Equation 2-15 which applies at or near the water surface or Equation 2-25 above. It should be recognized that Equation 2-25 yields a minimum weight and Equation 2-15 yields a median weight. Stone selection should be based on the weight gradations developed from each of the stone weights. The relative importance of these factors depends on the location of the structure and its elevation with respect to low water. When the toe protection is for scour caused by tidal or riverine currents alone, the designer is referred to EM 1110-2-1601. Virtually no data exist on currents acting on toe stone when they are a product of storm waves and tidal or riverine flow. It is assumed that the scour effects are partially additive. In the case of a revetment toe, some conservatism is provided by using the design stability number for toe protection in front of a vertical wall as suggested above.

(2) Suggested toe configurations. Guidance contained in EM 1110-2-1601 which relates to toe design configurations for flood control channels is modified for coastal revetments and presented in Figure 2-4. This is offered solely to illustrate possible toe configurations. Other schemes known to be satisfactory by the designer are also acceptable. Designs I, II, IV, and V are for up to moderate toe scour conditions and construction in the dry. Designs III and VI can be used to reduce excavation when the stone in the toe trench is considered sacrificial and will be replaced after infrequent major events. A thickened toe similar to that in Design III can be used for underwater construction except that the toe stone is placed on the existing bottom rather than in an excavated trench.

c. Seawalls and bulkheads.

(1) General considerations. Design of toe protection for seawalls and bulkheads must consider geotechnical as well as hydraulic factors. Cantilevered, anchored, or gravity walls each depend on the soil in the toe area for their support. For cantilevered and anchored walls, this passive earth pressure zone must be maintained for stability against overturning. Gravity walls resist sliding through the frictional resistance developed between the soil and the base of the structure. Overturning is resisted by the moment of its own weight supported by the zone of bearing beneath the toe of the structure. Possible toe configurations are shown in Figure 2-5.

(2) Seepage forces. The hydraulic gradients of seepage flows beneath vertical walls can significantly increase toe scour. Steep exit gradients reduce the net effective weight of the soil, making sediment movement under waves and currents more likely. This seepage flow may originate from general groundwater conditions, water derived from wave overtopping of the structure, or from precipitation. A quantitative treatment of these factors is presented in Richart and Schmertmann (1958).

(3) Toe apron width. The toe apron width will depend on geotechnical and hydraulic factors. The passive earth pressure zone must be protected for a sheet-pile wall as shown in Figure 2-6. The minimum width, B, from a geotechnical perspective can be derived using the Rankine theory as described in Eckert (1983). In these cases the toe apron should be wider than the product of the effective embedment depth and the coefficient of passive earth pressure for the soil. Using hydraulic considerations, the toe apron should be at least twice the incident wave height for sheet-pile walls and equal to the incident wave height for gravity walls. In addition, the apron should be at least 40 percent of the depth at the structure, d_s. Greatest width predicted by these geotechnical and hydraulic factors should be used for design. In all cases, undercutting and unraveling of the edge of the apron must be minimized.

(4) Toe stone weight. Toe stone weight can be predicted based on Figure 2-7 (from Brebner and Donnelly 1962)). A design wave between H_1 and H_{10} is suggested. To apply the method assume a value of d_t the distance from the still water level to the top of the toe. If the resulting stone size and section geometry are not appropriate, a different d_t should be tried. Using the median stone weight determined by this method, the allowable gradation should be approximately 0.5 to 1.5 W.

2-20. Filters

A filter is a transitional layer of gravel, small stone, or fabric placed between the underlying soil and the structure. The filter prevents the migration of the fine soil particles through voids in the structure, distributes the weight of the armor units to provide more uniform settlement, and permits relief of hydrostatic pressures within the soils. For areas above the waterline, filters also

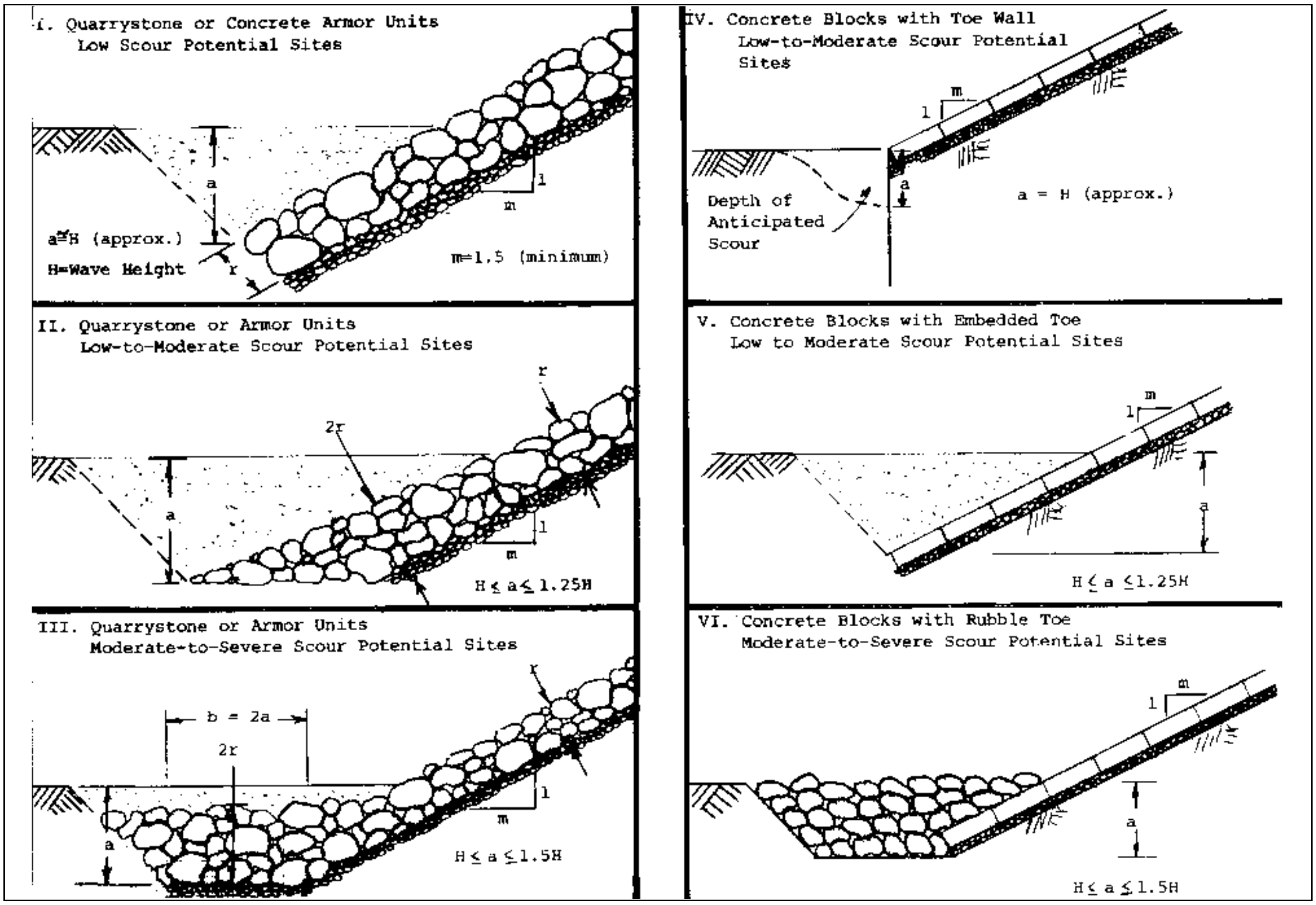

Figure 2-4. Revetment toe protection (Designs I through VI)

prevent surface water from causing erosion (gullies) beneath the riprap. In general form layers have the relation given in Equation 2-26:

$$\frac{d_{15\,upper}}{d_{85\,under}} < 4 \tag{2-26}$$

Specific design guidance for gravel and stone filters is contained in EM 1110-2-1901 and EM 1110-2-2300 (see also Ahrens 1981a), and guidance for cloth filters is contained in CW 02215. The requirements contained in these will be briefly summarized in the following paragraphs.

a. Graded rock filters. The filter criteria can be stated as:

$$\frac{d_{15\,filter}}{d_{85\,soil}} < 4 \text{ to } 5 < \frac{d_{15\,filter}}{d_{15\,soil}} \tag{2-27}$$

where the left side of Equation 2-27 is intended to prevent piping through the filter and the right side of Equation 2-27 provides for adequate permeability for structural bedding layers. This guidance also applies between successive layers of multilayered structures. Such designs are needed where a large disparity exists between the void size in the armor layer and the particle sizes in the underlying layer.

b. Riprap and armor stone underlayers. Underlayers for riprap revetments should be sized as in Equation 2-28,

$$\frac{d_{15\,armor}}{d_{85\,filter}} < 4 \tag{2-28}$$

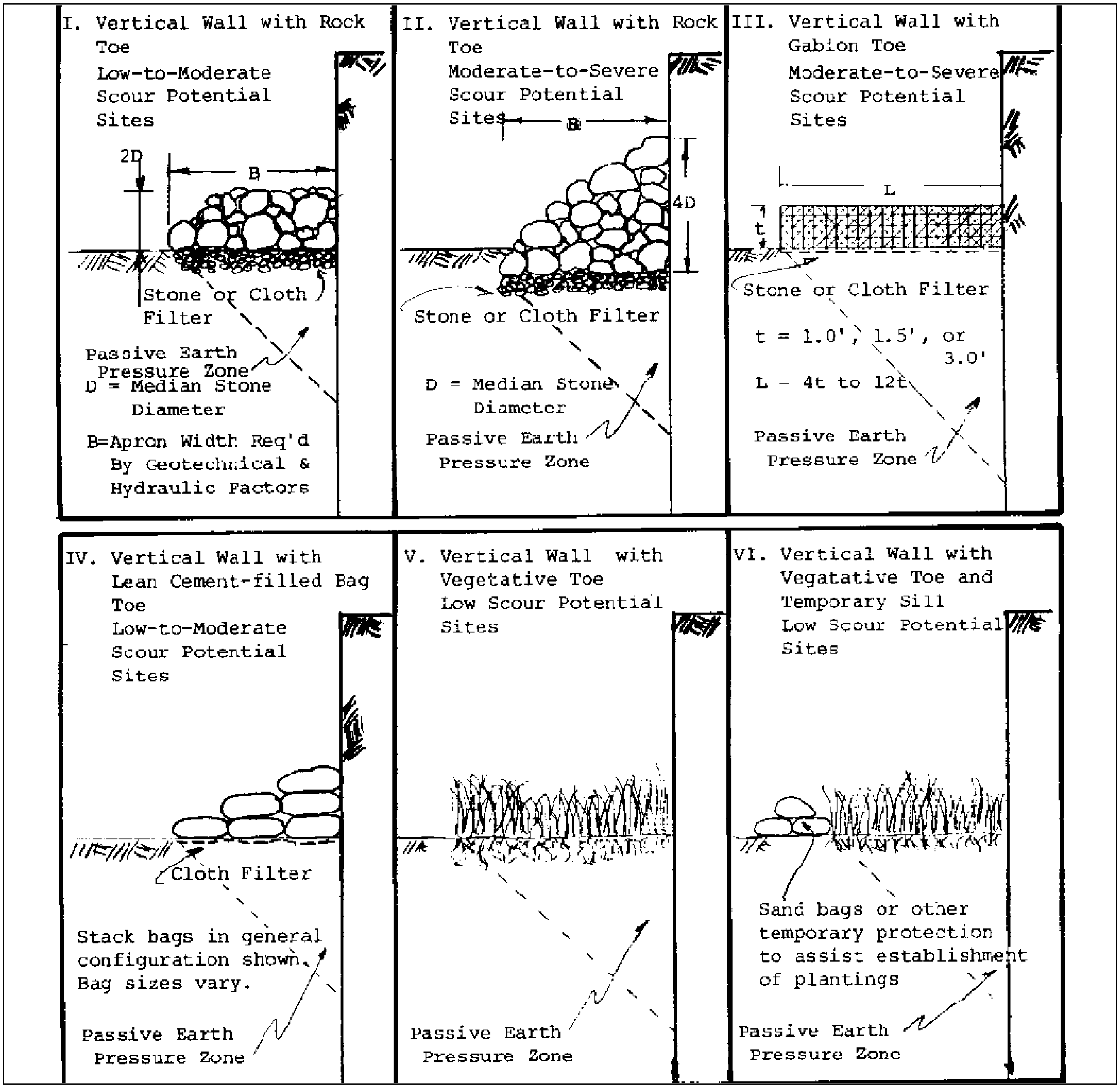

Figure 2-5. Seawall and bulkhead toe protection

where the stone diameter d can be related to the stone weight W through Equation 2-22 by setting n equal to 1.0. This is more restrictive than Equation 2-27 and provides an additional margin against variations in void sizes that may occur as the armor layer shifts under wave action. For large riprap sizes, each underlayer should meet the condition specified in Equation 2-28, and the layer thicknesses should be at least 3 median stone diameters. For armor and underlayers of uniform-sized quarrystone, the first underlayer should be at least 2 stone diameters thick, and the individual units should weigh about one-tenth the units in the armor layer. When concrete armor units with K_D > 12 are used, the underlayer should be quarrystone weighing about one-fifth of the overlying armor units.

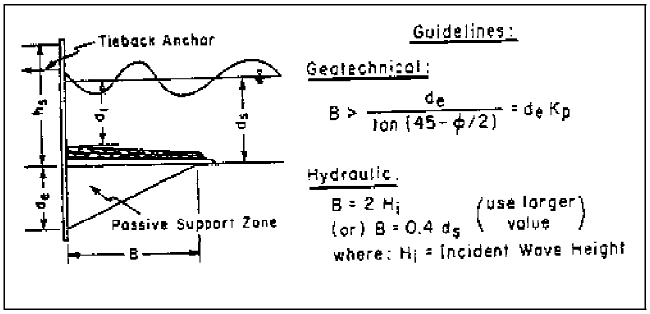

Figure 2-6. Toe aprons for sheet-pile bulkheads

c. Plastic filter fabric selection. Selection of filter cloth is based on the equivalent opening size (EOS), which is the number of the U.S. Standard Sieve having openings closest to the filter fabric openings. Material will first be retained on a sieve whose number is equal to the EOS. For granular soils with less than 50 percent fines (silts and clays) by weight (passing a No. 200 sieve), select the filter fabric by applying Equation 2-29:

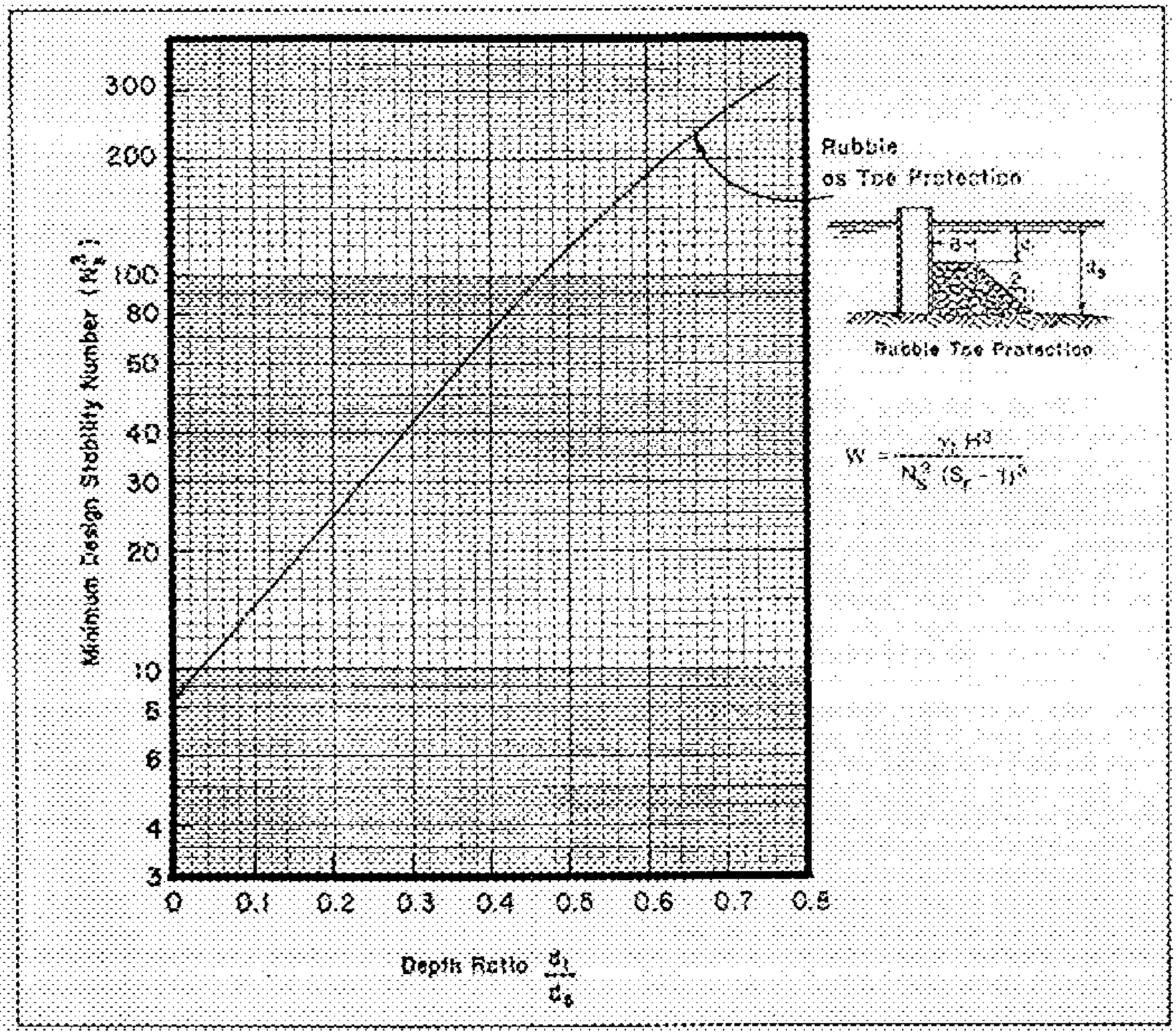

Figure 2-7. Value of N_s, toe protection design for vertical walls (from Brebner and Donnelly 1962)

$$\frac{EOS\ sieve}{d_{85\ soil}} \leq 1 \qquad (2\text{-}29)$$

For other soils, the EOS should be no larger than the openings in a No. 70 sieve. Furthermore, no fabric should be used whose EOS is greater than 100, and none should be used alone when the underlying soil contains more than 85 percent material passing a No. 200 sieve. In those cases, an intermediate sand layer may provide the necessary transition layer between the soil and the fabric. Finally, the gradient ratio of the filter fabric is limited to a maximum value of three. That is, based on a head permeability test, the hydraulic gradient through the fabric and the 1 in. of soil adjacent to the fabric (i_1) divided by the hydraulic gradient of the 2 in. of soil between 1 and 3 in. above the fabric (i_2) is:

$$Gradient\ ratio = \frac{i_1}{i_2} \leq 3 \qquad (2\text{-}30)$$

Studies such as those in Chen et al. (1981) suggest that these filter cloth selection requirements may be somewhat restrictive.

d. Filter fabric placement. Experience indicates that synthetic cloths can retain their strength even after long periods of exposure to both salt and fresh water. To provide good performance, however, a properly selected cloth should be installed with due regard for the following precautions. First, heavy armor units may stretch the cloth as they settle, eventually causing bursting of the fabric in tension. A stone bedding layer beneath armor units weighing more than 1 ton for above-water work (1.5 tons for underwater construction) is suggested (Dunham and Barrett 1974), and multiple underlayers may be needed under primary units weighing more than 10 tons. Filter guidance must be properly applied in these cases. Second, the filter cloth should not extend seaward of the armor layer; rather, it should terminate a few feet landward of the armor layers as shown in Figure 2-8. Third, adequate overlaps between sheets must be provided. For lightweight revetments this can be as little as 12 in. and may increase to 3 ft for larger underwater structures. Fourth, sufficient folds should be included to eliminate tension and stretching under settlement. Securing pins with washers is also advisable at 2-to 5-ft intervals along the midpoint of the overlaps. Last, proper stone placement requires beginning at the toe and proceeding up the slope. Dropping stone can rupture some fabrics even with free falls of only 1 ft, although Dunham and Barrett (1974) suggest that stones weighing up to 250 lb can safely be dropped from 3 ft. Greater drop heights are allowable under water where blocks up to 1 ton can be dropped through water columns of at least 5 ft.

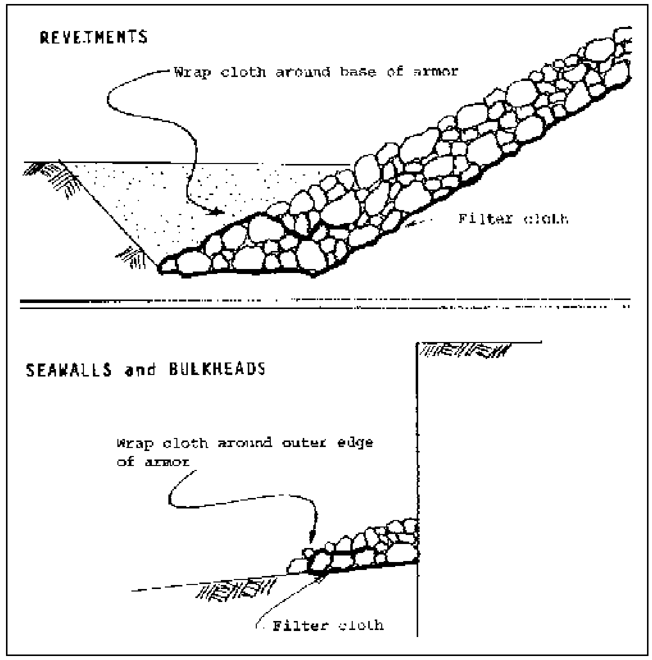

Figure 2-8. Use of filter cloth under revetment and toe protection stone

2-21. Flank Protection

Flank protection is needed to limit vulnerability of a structure from the tendency for erosion to continue around its ends. Return sections are generally needed at both ends to prevent this. Sheet-pile structures can often be tied well into existing low banks, but the return sections of other devices such as rock revetments must usually be progressively lengthened as erosion continues. Extension of revetments past the point of active erosion should be considered but is often not feasible. In other cases, a thickened end section, similar to toe protection, can be used when the erosion rate is mild.

2-22. Corrosion

Corrosion is a primary problem with metals in brackish and salt water, particularly in the splash zone where materials are subjected to continuous wet-dry cycles. Mild carbon steel, for instance, will quickly corrode in such conditions. Corrosion-resistant steel marketed under various trade names is useful for some applications. Aluminum sheetpiling can be substituted for steel in some places. Fasteners should be corrosion-resistant materials such as stainless or galvanized steel, wrought iron, or nylon. Various protective coatings such as coal-tar epoxy can be used to treat carbon steel. Care must always be taken to avoid contact of dissimilar metals (galvanic couples). The more active metal of a galvanic couple tends to act as an anode and suffers accelerated corrosion. The galvanic series of common metals in seawater is given in Table 2-6 (Uhlig 1971). This table can be used for estimating the corrosion potential of galvanic couples, but the complexity of corrosion processes makes it useful only as guide. For example, although aluminum and copper are closer together on the table than aluminum and stainless steel, in actual practice polarization effects with stainless steel make it more compatible with aluminum than aluminum copper couples. The Construction Engineering Research Laboratory (CERL) should be contacted when either performance or longevity is a significant requirement.

2-23. Freeze-Thaw Cycles

Concrete should be designed for freeze-thaw resistance (as well as chemical reactions with salt water), as concrete may seriously degrade in the marine environment. Guidance on producing suitable high quality concrete is presented in EM 1110-2-2000 and Mather (1957).

Table 2-6
Galvanic Series in Sea Water

	MATERIAL	MATERIAL (≈ ACTIVITY)
MORE	Magnesium	Stainless steel - 304 [AS]
ACTIVE	Stainless steel - 316 [AS]	
	Zinc	Lead
	Tin	
	Aluminum 52S4	
	Aluminum 4S	Magnesium bronze
	Aluminum 3S	Naval brass
	Aluminum 2S	
	Aluminum 53S-T	Nickel [AS]
	Yellow brass	
	Aluminum bronze	
	Red brass	
	Aluminum 17S-T	Copper, silicon bronze
	Aluminum 24S-T	
	Mild steel	Composition G bronze
	Wrought iron	Composition M bronze
LESS	Cast iron	Nickel [PS]
	Stainless steel-410 [AS]	
ACTIVE	Stainless steel-304 [PS]	
	Stainless steel-316 [PS]	

[AS] Active state
[PS] Passive state

2-24. Marine Borer Activity

Timber used in marine construction must be protected against damage from marine borers through treatment with creosote and creosote coal-tar solutions or with water-borne preservative salts (CCA and ACA). In some cases, a dual treatment using both methods is necessary. Specific guidance is included in EM 1110-2-2906.

2-25. Ultraviolet Light

The ultraviolet component of sunlight quickly degrades untreated synthetic fibers such as those used for some filter cloths and sand-bags. Some fabrics can completely disintegrate in a matter of weeks if heavily exposed. Any fabric used in a shore protection project should be stabilized against ultraviolet light. Carbon black is a common stabilizing additive which gives the finished cloth a characteristic black or dark color in contrast to the white or light gray of unstabilized cloth. Even fabric that is covered by a structure should be stabilized since small cracks or openings can admit enough light to cause deterioration.

2-26. Abrasion

Abrasion occurs where waves move sediments back and forth across the faces of structures. Little can be done to prevent such damages beyond the use of durable rock or concrete as armoring in critical areas such as at the sand line on steel piles.

2-27. Vandalism and Theft

At sites where vandalism or theft may exist, construction materials must be chosen that cannot be easily cut, carried away, dismantled, or damaged. For instance, sand-filled fabric containers can be easily cut, small concrete blocks can be stolen, and wire gabions can be opened with wire cutters and the contents scattered.

2-28. Geotechnical Considerations

The stability of vertical bulkheads, particularly sheet-pile structures, requires consideration of overturning and stabilizing forces. Static forces include active soil and water pressures from the backfill, water and passive soil pressures on the seaward side, and anchor forces (when applicable). Dynamic forces are the result of wave action and seepage flow within the soil. Wave impacts increase soil pressure in the backfill and require larger resisting passive earth pressures and anchor forces to ensure stability. Seepage forces reduce passive pressures at the toe and tend to decrease factors of safety. Toe scour decreases the effective embedment of the sheetpiling and threatens toe stability of the structure. This scouring action is caused by currents along the bottom and by pressure gradients. Both of these are induced by waves on the surface. A quantitative treatment of these geotechnical considerations can be found in Richart and Schmertmann (1958).

2-29. Wave Forces

Wave forces are determined for cases of nonbreaking, breaking, or broken waves. These cases are dependent on the wave height and depth at the structure. Wave forces for a range of possible water levels and wave periods should be computed.

a. Nonbreaking waves. Current design methods apply to vertical walls with perpendicularly approaching wave orthogonals. The Miche-Rundgren method as described in the SPM should be used. Curves are given in Chapter 7 of the SPM for walls with complete or nearly complete reflection. Complex face geometries cannot be handled, but methods are described which can be used in some cases to correct for low wall heights (where overtopping occurs), oblique wave attack on perpendicular structure faces, and walls on rubble bases.

b. Breaking waves. Breaking waves on vertical structures exert high, short-duration impulses that act in the region where the wave hits the structure. The method developed by Minikin as described in the SPM is recommended, particularly, for rigid structures such as sheet-pile structures or concrete gravity-type structures with pile supports. The Minikin method can yield extremely high wave forces compared to nonbreaking waves. This sometimes requires the exercise of proper judgment by the designer. Curves are given in the SPM to correct for low wall heights. For semirigid structures such as gravity-type seawalls on rubble foundations Equation 2-31 is recommended. Equation 2-31 was developed from Technical Standards for Port and Harbour Facilities in Japan (1980).

$$F = \frac{1}{2}\left[d_s\left(P_1 + P_2\right) + h_c\left(P_1 + P_4\right)\right] \qquad (2\text{-}31)$$

The total force, F, per unit length of the structure, includes both the hydrostatic and dynamic force components. Figure 2-9 illustrates the pressure distribution on the face of the structures due to the breaking waves. The key pressure components can be determined by:

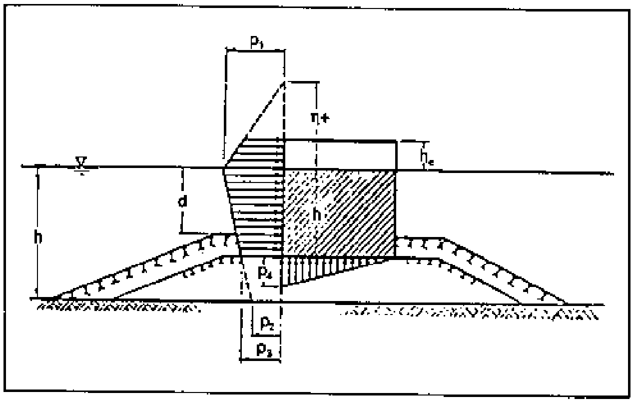

Figure 2-9. Breaking wave pressures on a vertical wall

$$P_1 = (\alpha_1 + \alpha_2)\gamma_w H_b \quad (2\text{-}32)$$

$$P_3 = \alpha_3 P_1 \quad (2\text{-}33)$$

$$P_4 = \left(1 - \frac{h_c}{1.5\,H_b}\right) P_1 \quad (2\text{-}34)$$

where

$$\alpha_1 = 0.6 + \frac{1}{2}\left[\frac{4\pi h/L}{\sinh\,(4\pi h/L)}\right]^2 \quad (2\text{-}35)$$

$$\alpha_2 = \min\left[\left(\frac{h_b - d}{3h_b}\right)\left(\frac{H_b}{d}\right)^2 , \ \frac{2d}{H_b}\right] \quad (2\text{-}36)$$

$$\alpha_3 = 1 - \frac{d_s}{h}\left[1 - \frac{1}{\cosh\left(\frac{2\pi h}{L}\right)}\right] \quad (2\text{-}37)$$

where

γ_w = specific weight of water

h_c = height of crest of caisson above swl

d = depth at top of rubble mound

d_s = depth at base of caisson

H_b = highest of the random waves breaking at a distance of $5H_s$ seaward of the structure; H_s is the significant wave height of the design sea state

h_b = water depth where H_b is determined

h = water depth at toe of compound breakwater

L = wave length calculated by linear wave theory at the structure for wave period of H_s

As an example, for a vertical wall, 4.3 m (14 ft) high sited in sea water with d_s = 2.5 m (8.2 ft) on a bottom slope of 1:20 (m = 0.05) and experiencing wave crests at an interval of 10 sec, the force on the wall would be determined as follows:

Since there is no rubble-mound base, the water depth d_s = 2.5 m. Using a wave period T = 10 sec and Figure 7-4 of the SPM, the breaking wave height, H_b, is found to be 3.2 m (10.5 ft). Without knowledge of the significant wave height, H_s, the breaking depth, h_b, is determined directly by using SPM Figure 7-2, which yields h_b = 3.07 m (10 ft). The wave breaks at a distance of 11.4 m (37 ft) [(3.07 - 2.5)/0.05] from the wall. Using SPM Appendix C Table C-1, wave length, L, at d_s = 2.5 m is determined to be 48.7 m (160 ft). Then, α_1, α_2, and α_3 are calculated to be 1.036, 0.101, and 0.950, respectively. Crest height, h_c, is less than 1.5 H_b (1.8<4.8) and overtopping exists. The pressure components P_1, P_3, and P_4 are computed from the above equations to be 36.4 kN/m^2 (1,742.8 lb/ft^2), 34.6 kN/m^2 (1656.6 lb/ft^2), and 22.8 kN/m^2 (1,091.7 lb/ft^2), respectively. Equation 3-31 yields a total horizontal force due to the breaking wave of 142 kN/m^2 (6,799 lb/ft^2).

c. Broken waves. Some structures are placed in a position where only broken waves can reach them. In those cases approximate broken wave force, F, per unit length of structure can be estimated (Camfield 1991) by Equation 2-38:

$$F = 0.18\ \gamma\ H_b^2 \left(1 - \frac{X_1\ m}{R_A}\right)^2 \quad (2\text{-}38)$$

where γ is the specific weight of water and m is the beach slope (m=tan θ). Other variables of Equation 2-38, H_b, X_1, and R_A are defined in Figure 2-10. The adjusted

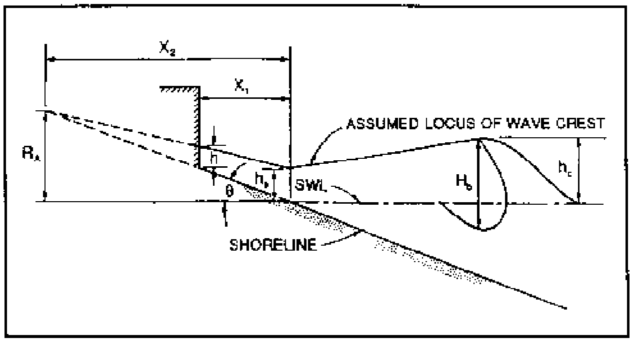

Figure 2-10. Wave pressure from broken waves

wave runup height, R_A, which would occur if the wall was not present can be determined by using Equation 2-6 (rough slopes) or following the methods described in Chapter 2-13 for smooth slopes or slopes covered with rubble other than quarrystone. If accurate force estimates are needed, model tests are required.

For example, deepwater waves are H_{mo} = 0.91 m (3 ft) and T_p = 12 sec. The waves cross 3.05 m (10 ft) of cobble shoreline with a slope of m = 0.10 before impacting on a wall. From Figure 7-3 in SPM (1984), breaking wave height H_b is 2.05 m (6.75 ft). Using Equation 2-7 we find ξ = 1.57, and Equation 2-6 yields R_{max} = 1.36 m (4.48 ft). Use R_{max} for the adjusted runup, R_A, in Equation 2-38 to find the force per unit length of wall is 4.58 kN/m length of wall (317 lb/ft length of wall).

2-30. Impact Forces

Impact forces constitute an important design consideration for shore structures because high winds can propel small pleasure craft, barges, and floating debris and cause great impact forces on a structure. If site or functional conditions require the inclusion of impact forces in the design, other measures should be taken to limit the depth of water against the face of the structure by providing a rubble-mound absorber against the face of the wall or a partly submerged sill seaward of the structure that will ground floating masses and eliminate the potential hazard. In many areas impact hazards may not occur, but where the potential exists (as for harbor structures), impact forces should be evaluated from impulse-momentum considerations.

2-31. Ice Forces

a. General. Ice can affect marine structures in a number of ways. Moving surface ice can cause significant crushing and bending forces as well as large impact loadings. Vertical forces can be caused by the weight of ice on structures at low tide and by buoyant uplift at high tide of ice masses frozen to structural elements. EM 1110-2-1612 should be reviewed before designing any structure subject to ice forces.

b. Damages. Ice formations can cause considerable damage to shoreline at some points, but their net effects are largely beneficial. Spray “freezes” on banks and structures and covers them with a protective layer of ice. Ice piled on shore by wind and wave action does not generally cause serious damage to beaches, bulkheads, or protective riprap, but it provides additional protection against severe winter waves. Some abrasion of timber or concrete structures may be caused, and individual members may be broken or bent by the weight of the ice mass. Piling is sometimes slowly pulled by the repeated lifting effect of ice frozen to the piles or attached members, such as wales, and then it is forced upward by a rise in water stage or wave action. Superstructure damages also sometimes occur due to ice.

2-32. Hydraulic Model Tests

The guidance contained in this manual is suitable for preliminary design of all coastal structures and for final design of minor or inexpensive works where the consequences of failure are not serious. For most cases, however, the final design should be verified through a model testing program. Design deficiencies can be identified with such models, and design economics may be achieved which more than offset the cost of the study. Hudson et al. (1979) contains information on current hydraulic modeling techniques.

2-33. Two-Dimensional Models

Two-dimensional tests are conducted in wave tanks or flumes. Such tests are useful for evaluating toe stone and armor stability, wave runup heights, and overtopping potential. Generated waves may be either monochromatic or irregular depending on the capabilities of the equipment. Monochromatic waves represent the simplest case, and they form the basis for the majority of current design guidance. Irregular waves, on the other hand, are a closer representation of actual prototype conditions. Their use, however, adds to the complexity of a modeling program.

2-34. Three-Dimensional Models

Three-dimensional models are built in large shallow basins where processes such as wave refraction and diffraction are of interest. They can also lead to qualitative results for sediment transport studies. However, these issues are generally unimportant for the design of revetments, seawalls, and bulkheads; therefore, the use of three-dimensional models would be unusual for such structures.

2-35. Previous Tests

WES has conducted a number of two- and three-dimensional model studies of site-specific projects. Details on five of these are given below. Units are given in prototype dimensions.

a. Fort Fisher NC (1982). Important features were (Markle 1982):

Scale	1:24
Waves	Heights of 5.5 to 17.2 ft Periods of 8, 10, and 12 sec
Depths	12, 14.7, 17, and 19 ft
Revetment slope:	1:2

The toe consisted of 8,919-lb StaPods on bedding stone. The sizes of the armor units were 5,900 lb (specially placed) and 8,900 lb (randomly placed). These were stable and undamaged in depths to 14.7 ft. At depths of 17 and 19 ft, considerable damages were experienced, but no failures occurred.

b. El Morro Castle, San Juan, PR (1981). Important features were (Markle 1981):

Scale	1:38.5
Waves	Heights of 10 to 23.3 ft Periods of 15 and 17 sec (north revetment)
	Heights of 2.5 to 10.5 ft Periods of 9, 15, and 17 sec (west revetment)
	18 and 19.9 ft (north revetment)
	13 and 14.9 ft (west revetment)
Revetment slope:	1:3

The toe protection was generally a 10-ft-wide armor stone blanket except in certain areas of the north revetment where a low-crested breakwater was used. Armor stone sizes were 10,300 lb (west revetment), 24,530 lb (north revetment), and 9,360 lb (north revetment behind breakwater). All armor stone was randomly placed.

c. Generalized harbor site for the U.S. Navy (1966). Important features were (USAEWES 1966):

Scale	1:15
Waves	Heights of 5, 10, 15, and 20 ft 10-sec periods
Depths	20 to 40 ft
Revetment slope:	1:5

No toe protection was provided (the toe extended to the flume bottom). Stable rock sizes and values of K_d were reported for several wave conditions.

d. Railroad fills at Ice Harbor and John Day Reservoirs (1962). The tests were conducted for both riprap stability and runup. Important features were (USAEWES 1962):

Scale	1:12
Waves	Height of 2.4 to 2.6 ft Periods of 3, 4, 5, 6, and sec
Depths	20 to 40 ft
Revetment slope:	1:2

No toe protection was provided. The stable W_{50} sizes were

W_{50}	H
300 lb	3.0 to 3.4 ft
500 lb	2.0 to 4.1 ft
700 lb	3.9 to 4.9 ft

e. Levees in Lake Okeechobee, FL (1957). The tests were conducted for both wave runup and overtopping. Important features were (USAEWES 1957):

Scale	1:30 and 1:17
Waves	Heights of 4, 6, 8, and 12 ft Periods of 4.5 to 7 sec
Depths	10, 17.5, and 25 ft

Revetment slope: 1:3, 1:6, and
composite slopes

No toe protection was considered. The tests produced a series of runup and overtopping volume curves.